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# PROCEEDINGS

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# DISCUSSION
# OF PROCEEDINGS - SEPARATES

148, 166

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## DISCUSSION OF BANK STABILIZATION BY REVETMENTS AND DIKES PROCEEDINGS-SEPARATE NO. 148

HARRISON V. PITTMAN,[4] M. ASCE.—Timely and interesting is this description of bank stabilization along the Lower Mississippi River, presented in the interest of both navigation and flood control. However, the paper omits the many interesting details of the historical development of the various plans, types of structures, devices, and materials that have been either tried and abondoned or else adopted and improved upon during the many years of effort to stabilize and control the Mississippi River. Statements concerning their abandonment or adoption, together with the specific reasons for such decisions, would be valuable and enlightening contributions to a study of this river. For example, an asphaltic type mattress was once developed and used experimentally by the New Orleans District, Corps of Engineers, United States Department of the Army. This project showed great promise and received world-wide publicity, but eventually it was abandoned.

A paper by Charles Senour, M. ASCE., gives an excellent presentation of the historical and physical sequences of flood control and channel regulation in the Lower Mississippi River valley.[5] Mr. Senour's paper serves as a valuable reference for a fuller understanding of the subject matter of the authors' paper.

The problem of bank stabilization and the improvement of the navigable channel of the Mississippi River is an old one. There was on file in the Office of the District Engineer, St. Louis, Mo. (about 1911), a drawing which bore the signature of Robert E. Lee, First Lieutenant, Corps of Engineers, U. S. Army, and was dated 1835. It depicted a design for a number of spur dikes, consisting of stone and brush, which were installed along the waterfront of St. Louis to improve navigation. These dikes probably were among the very first regulatory structures to be installed on the Mississippi River. Throughout succeeding years various plans have been tried, and experimentation and studies in field and office will be continued in order to improve methods and materials for a more satisfactory and permanent control of the river.

Records show that the Mississippi is a river with caving banks and shifting bottom. No other river under improvement for purposes of navigation equals it in the magnitude of its bed disturbance. This is particularly true of the Lower Mississippi River, although partly applicable to the Middle Mississippi River.

The nearest approach to completion of open river regulation on the Mississippi River, by means of contraction works for rectifying and deepening the channel and by revetments for stabilizing the banks, is that which has been accomplished between St. Louis and Cairo. However, the plan of improvement, alike in principle to that now in use on the Lower Mississippi River, makes no provision for the rectification and stabilization of the uncontrolled, erosive alluvium bed of the river with its variable slopes, velocities, pools, and

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[5] "New Project for Stabilizing and Deepening Lower Mississippi River," by Charles Senour, *Transactions*, ASCE, Vol. 112, 1947, p. 277.

shoals. It is this lack of complete control over the entire cross section of the river that causes much of the severe damage to (or the destruction of) many of the stabilization structures. The lack of bed control constitutes the major part of the complex and difficult problem of permanently stabilizing the river at the present time.

In the writer's opinion, the only feasible plan that has been proposed for the complete regulation of the Mississippi River was that developed by the late W. M. Penniman, M. ASCE.[6]

This plan proposed three forms of permanent construction: (1) The necessary side contraction, (2) the stabilizing of all concave bends, and (3) the regulation of the entire bed of the stream by fixation of the crests of the controlling bars. This fixation of natural or artificial bars would equalize the fall of the river and preserve the required cross section for complete regulation. Sill dams or cross weirs would regulate the flow, thus maintaining the desired depth of channel obtained by side contraction and bank protection.

Two forms of construction that were in use at the time Mr. Penniman's plan was proposed were considered to be standard. These were side contraction and the stabilization of concave bends. The third form of construction mentioned in the plan remains untried on any major stream in the United States.

*Instability of the River Banks.*—The different types of caving banks encountered on the Mississippi River which are of constant concern to engineers, have been classified and described.[7] Following surveys and field observations extending over a period from about 1877 to 1908, it was decided to classify caving banks as eroding banks, slumping banks, sinking banks, and sliding or slipping banks.

An eroding bank is one in which the whole side of the river bank, from the water surface to the foot of the bank on the river bottom, is gradually wearing away under the scour of the moving water. Its soil is not cohesive enough to withstand the attack of the ordinary currents natural to the mean-water and low-water stages of the river. The bank becomes undercut and the upper parts break away. Stopping the erosion below the low-water line is of major importance. The slumping bank is usually steep and perhaps vertical on the side next to the river. It becomes dry and perhaps cracked by evaporation during a continued low-water season and becomes thoroughly soaked and filled with water during high waters. As the water falls and the temporary support given it by the water pressure is taken away, the front breaks off from the dryer or tougher sections of the bank behind, and it slides down into the bed of the river. The sinking bank is one in which a large mass of earth material rests upon a layer of very soft material which may be squeezed out by pressure of the overlying earth or washed out. In such cases, the entire river bank may settle uniformly, or nearly so, for any depth from a few inches to many feet, its top surface remaining nearly horizontal. The stratum of quicksand or other soft material may be many feet below the ordinary water

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[6] *House Document No. 50*, 61st Cong., 1st Session, 1909, Appendix No. 5.

[7] *House Document No. 50*, 61st Cong., 1st Session, 1909, Appendix No. 1.

surface of the river, and such sinking banks are therefore liable to be developed at any stage of water.

The sliding or slipping bank is one where a large mass of material slides down the bank into the river. Usually, the slide results from the fact that material of considerable weight rests upon a smooth inclined surface of slippery material, and under the influence of heavy rainfall the earth mass slides as a unit until its foot reaches some solid point of support farther down the bank. Slides are usually entirely independent of high water and dependent almost entirely on heavy rainfall and poor runoff back of the river bank.

As explained, sinking banks and sliding banks are mainly independent of river conditions, and slumping banks occur only intermittently at high water, but eroding banks, especially below low water, are constantly in action and, therefore, are to be feared the most. These different types of caving banks must be considered individually when designing works for their most effective and permanent stabilization.

The authors state, under the heading, "Revetments: Design of Revetments," that "Experience has shown that, to be effective, a revetment***must extend from the top of the bank to the toe of the underwater slope***." Normally, it should extend a short distance beyond the thalweg of the stream.

Under the heading, "Dikes, Groins, and Retards," no mention is made as to whether these structures are now used for the contraction of the river to some predetermined width for increasing the low-water navigable depths over the bars or crossings to 12 ft, or as training works for guiding the flow into the next bend below.

The statement that "Bank protection dikes are spaced from 1½ times to2½ times the length of the upstream dike * * *" (see under the side heading, "Pile Dikes") is not quite clear. Referring to Fig. 4, the distance of the second dike downstream from the first dike does not conform to this rule. The rule of thumb in general use would be to locate the second dike downstream from the "foot" of the first dike, a distance of from 1½ times to 2½ times the distance between the foot of the first dike and the high bank, measured on a line perpendicular to the bank. Then, the third dike would likewise be located with reference to the second dike, and so on for the remaining dikes in the system. This fact applies to all dikes inclined downstream and is based on the effectiveness of dikes in deflecting the currents away from the bank. Should the dikes be placed perpendicular to the bank, the spacing of each succeeding dike applies to the actual length of the preceding dike.

The authors state, under the heading, "Problem of Bank Stabilization: Causes of Failure," that

> "As of January, 1950, there were approximately 170 miles of stabilized banks on the Lower Mississippi River between Cairo and Baton Rouge, La. Approximately 255 miles of caving banks remain to be stabilized."

Mr. Senour[5] writes that

> "The study developed that, in the 737 miles between Cairo and Baton Rouge, 97½ miles of effective bank revetment were already in place, and indicated that, to stabilize the banks between the two points, about 230 additional miles would be required***."

A comparison of these two statements indicates that between February, 1946, and January, 1950, an additional 72½ miles of bank were stabilized, but there still remained 255 miles to be stabilized—25 miles more than was required in 1946. At this rate, it is rather difficult to estimate just when this project will be completed and what the cost will be.

Under the heading, "Problem of Bank Stabilization: Causes of Failure," the authors state that

> "Because of the great depths, swift currents, and opaqueness of the water, determinations from subaqueous examinations of mattress work by divers have not been entirely satisfactory."

This is a questionable statement since, in 1932, a diver was employed to determine the condition of the thirty-year-old willow fascine mattress placed along the waterfront at Helena (Ark.). The diver's examination showed that the mattress soon would reach the limit of its effectiveness, although no serious scouring action or caving had as yet developed. Samples of the tie wires appeared to be of bronze, still intact, and effective. The top poles, cribbing, almost all the stone ballast, and the greater part of the brush composing the facines had disappeared. The brush that remained in the fascines had been eroded to about the size of a finger. As a result of these observations, a new willow fascine mattress was installed within a short time.

The authors assert, under the heading, "Problem of Bank Stabilization: Causes of Failure," that

> "Dikes and other timber structures are ordinarily satisfactory when material deposits from within three to four years. Until such deposits form, the dikes are subject to destruction by drift, ice, and vibration. If no accretion forms, the dikes are subjected to deterioration by decay. Experience indicates that no accretionary deposit is more permanent than the structure that produces it; therefore, retards and dikes are constructed principally of creosoted timbers to give longer life."

These statements are concurred in as they call attention to the temporary nature of these wood structures even when they have been creosoted and the required deposit has been obtained. It continues to be necessary to maintain effectively these structures in order to hold the artificial accretion, or else to place revetments along the river face of such an accretion so as to insure its permanency. The use of concrete piles would appear to bear investigation because of the relative permanency of the concrete, the ease of manufacture and placing, and the possible economy. The writer knows of no instance where concrete piles have been used or even considered in works for open river regulation on the Mississippi River.

The placement of revetments in isolated localities for the purpose of safeguarding front-line levees from destruction by caving banks has always been an expedient of a temporary nature. Rarely can such work be depended on to fit in with, and become an integral part of, the continuous, systematic, and more permanent stabilization works. Their cost constitutes a continued expenditure, in excess of the funds normally required for the comprehensive plan of improvement of the river for navigation and flood control, and prolongs the time for its completion.

Reference is made (under the heading, "Problem of Bank Stabilization: Experimental Work") to the three experimental revetments installed in 1949, "***to support revetments in the opposite bends immediately below." Information is desired as to whether any training works were installed below the foot of the upstream revetment so as to direct the low-stage and mid-stage flows over the crossing between the upper and lower revetments. This installation would bring about a deepening and a fixation of such crossing and would establish a satisfactory approach flow into, and a parallel flow alongside of, the downstream revetment. Such treatment is imperative to avoid downstream migration of both bar and crossing, to maintain a navigable depth, and to insure the integrity of the mattress in the bend immediatley below by preventing masking or destruction by shifting attacks of flow.

E. R. de la Sayette,[8] J.M. ASCE.—In their interesting paper, Messrs. Haas and Weller state, under the heading, "Revetments: Articulated Concrete Mattress," that "***a less permeable modification of the articulated concrete mattress, a so-called V-type mattress***," has been developed. It might be of interest to American civil engineers to know that this type of revetment, which has not been used extensively in the United States, is now being used in France on two bank stabilization projects.

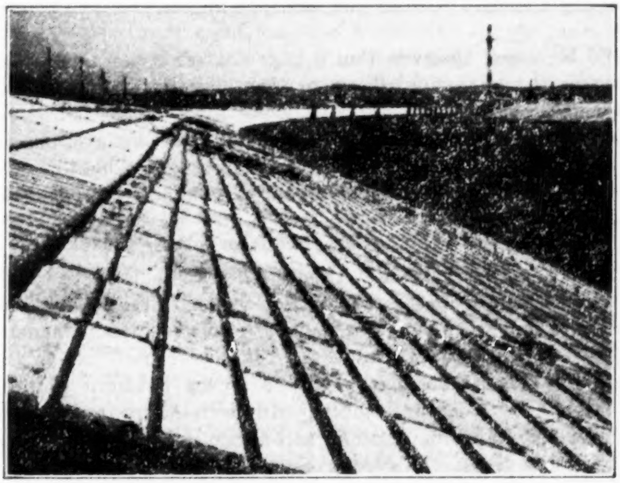

Fig. 8.—Revetment Along the Donzère-Mondragon Canal in France

The first of these is shown in Fig. 8. This is the revetment on part of the 17-mile Donzère-Mondragon Canal, one of the main features of the Rhône River development. The second is the revetment on the Randens Canal, the tailrace channel of the Isère-Arc hydroelectric project in the French Alps.

These two enterprises illustrate that articulated concrete mats can be an economical solution for the revetment of canal banks of small area, as well as the large bank areas of the Mississippi River.

[8] Head Engr., Société du Vacuum Concrete, Paris, France.

The solution to the bank stabilization problem, as described by the authors, has been carefully studied by the writer, who paid a valuable visit to the Memphis District of the Corps of Engineers, United States Department of the Army, in 1951. However, solutions must be chosen according to the scale of the individual project. At Donzère, France, the project involves 6,000,000 sq ft of revetment. Each revetment unit, called a "square," is 3 ft wide by 24 ft long; it is composed of 22 individual blocks, 3 $\frac{3}{16}$ in. thick. At Randens, France, the area covered is 250,000 sq ft, the total length of the canal being only 1,500 yds. The square in this case is 3 ft wide and 18 ft long, made up of 14 blocks, 4¾ in. thick.

The casting and handling methods are similar for the two projects, both making use of vacuum concrete processes. The concrete is poured on concrete casting beds. For each square the side form is composed of a concrete curb with steel angles set in the casting bed to obtain the bevel at the bottom of the V-shape. The fresh concrete is screeded and the upper bevel of the V is obtained by stamping this concrete (under vibration) with steel angles protruding from the bottom of the semi-rigid vacuum mat. The fresh concrete is then vacuum processed and the mat immediately re-used on the next square.

Unmolding, placing in stock, removing from stock, and even the placing on canal banks is done by using a flexible vacuum lifter, invented by K. P. Billner and experimented with by the Memphis District of the Corps of Engineers.

It should be noted, however, that a huge floating construction plant such as that used on the Mississippi River, or even a smaller but similar machine, could not be operated economically on small jobs. The squares described were placed one at a time at Randens, and two at a time at Donzère. The lifter used on these projects was found to be quite adaptable for placing the squares, since it can be used under water with minor adjustments to the equipment.

Serge Leliavsky,[9] M. ASCE.—Credit is due the authors of this interesting paper for having re-opened discussion on some vital problems of river-training engineering—such as the old debate between the defenders of the "passive" revetments system and those who favor the "active" spur, or groin, system. These problems were once in the forefront of progressive hydraulic interest, but they were subsequently eclipsed by the spectacular applications of the Prandtl-von Kármán turbulence theory, which monopolized attention for many years. This trend resulted in the almost complete neglect of the three-dimensional aspects of the sediment transportation problem as evolved by the river-training experts of the first decades of the twentieth century. Instead, a characteristically two-dimensional approach became prevalent, marked by a galaxy of brilliant attempts to find new significant parameters correlating the turbulence theory with the results of laboratory tests on sediment transportation, instead of paying more attention to the three- dimensional velocity observations in natural rivers.

[9] Civ. & Hydr. Engr., Maadi, Cairo, Egypt.

It is true that these two-dimensional attempts may clarify a number of problems relevant to sediment transportation in general. However, they will not yield the basis for solving the problem of sediment regimen in rivers because, as the ratio of channel width to channel depth begins to increase, the water flow and sediment movement—even in the rigid flume of a laboratory experiment[10]—cease to follow routes that are even approximately parallel, and become distinctly helicoidal; that is, basically three-dimensional. The well-known experiments of Vito A. Vanoni,[10] M. ASCE, are a case in point. In spite of all the precautions taken to produce parallel flow, in the experiments, the water would not spread the sediment uniformly over the width of the steel flume, but, instead, transported it in the form of three clouds or ribbons, parallel to the flow, which settled in three clearly defined streaks when the flow was stopped. This result could be explained only by the action of secondary circulation—that is, four helicoidal currents.

If conditions such as these prevail in an artificially-built, rigid, rectangular channel, the three-dimensional effect in a naturally winding river must be much greater. The classical theory[11] of James Thomson, accepted subsequently by Hubert Engels, Max Möller, and others, lends support to the foregoing statement by showing that the meanders themselves, and the shape of the channel in them, are consequences of a naturally created three-dimensional current.

Therefore, the advantages of a group of spurs or groins, as opposed to those of a revetment for an equal length of bank—and the optimum layout for the former system—must be judged by the effect on the flow lines, particularly the helicoidal current, controlling the formation of the earthen channel.

Gerard H. Matthes,[12] Hon. M. ASCE, states that he was unable to observe the helicoidal current in the Lower Mississippi River, but this statement must have resulted from his method of observation. One would not expect the three-dimensional effect in a river the size of the Mississippi to be so obvious and easily observed by the naked eye as were the sediment ribbons in Mr. Vanoni's trough. Conclusions on the existence and effect (if any) of the helicoidal current in a river of this size must be based on instrumental observations of the direction of the flow, at different stations along the channel, and at various depths.

Therefore, a rational reply to the main problem in the authors' paper—that of the relative advantages of spurs as opposed to revetments—must be based on observations of the regimen of velocities in the zone affected.

As to the writer's knowledge, two different apparatuses have been used for this purpose—a rather light instrument applied by B. O. Hellstrom to the study of helicoidal flow in small canals,[12] and a much larger, more advanced device used by Nicolas de Leliavsky,[13,14] in the design of training works on the Dnieper River.

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[10] "Transportation of Suspended Sediment by Water," by Vito A. Vanoni, *Transactions*, ASCE, Vol. 111, 1946, p. 67.

[11] *Proceedings*, Royal Soc. of London, Vol. 25, 1877, p. 6.

[12] "Macroturbulence in Natural Stream Flow," by Gerard H. Matthes, *Transactions*, Am. Geophysical Union, Vol. 28, Pt. 2, 1947.

[13] "Des Courants Fluviaux et de Formation du Lit Fluvial," by Nicolas de Leliavsky, *Proceedings*, Sixth International Navigation Cong., The Hague, Holland, 1894.

[14] "Résultats obtenus par le Dragage sur les Seuils des Rivières," by Nicolas de Leliavsky, *Proceedings*, Eighth International Navigation Cong., Milan, Italy, 1905.

As seen from Fig. 9, the apparatus of Mr. de Leliavsky consisted of a float built of two barges about 56 ft in length, with a tripod that rested on the bed of the river during the observations. A current meter of the Amsler type, with a tail extension similar to the tail of an airplane, was attached by means of a universal articulation to the tripod, and connected with indicators that could be read from an overhead platform. The tower and tripod were mounted amidships and the winch for raising the tripod and meter was near the stern. The barges were securely anchored before the tripod was lowered. It is essential to realise that in spite of the turbulent fluctuations, the apparatus yielded consistent average velocities for all the points observed. An example of a set of such observations is given in Fig. 10.

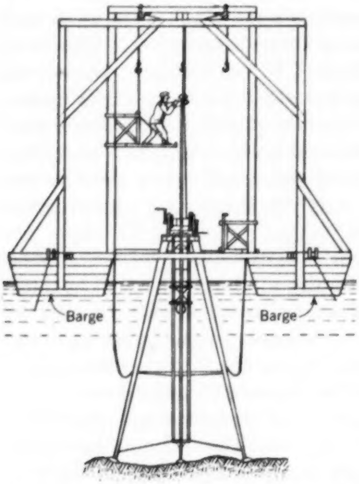



FIG. 9.—FRONT VIEW OF THE CURRENT MEASURING APPARATUS OF MR. DE LELIAVSKY

The empirical law derived by Mr. de Leliavsky from a large number of such observations correlated the depth with the convergency of the flow lines.

Considering these results, uniformity of river flow is but an academic assumption, which is true in the statistical sense only—that is, from the standpoint of over-all averages such, for instance, as the total discharge of the river. In nature, however, a river channel is a mosaic composed of individual small spots in which the flow is either permenently accelerated or permanently decelerated. This condition does not interefere with statistical over-all bulk

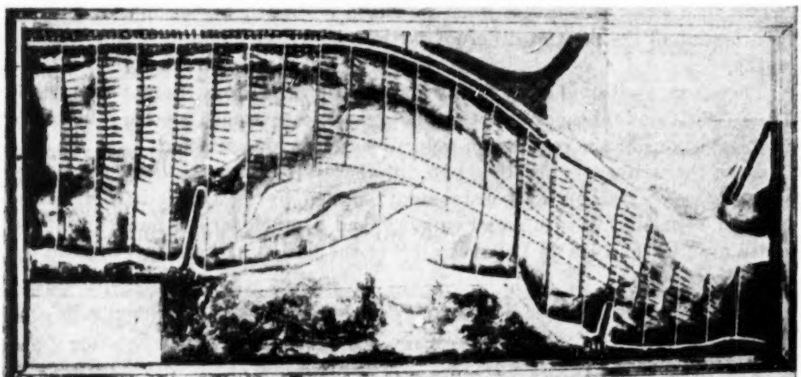



FIG. 10.—A SECTION OF THE DNIEPER CHANNEL, SHOWING THE VELOCITIES RECORDED BY MR. DE LELIAVSKY, IN 1903

uniformity because in the first case the flow lines are convergent and in the second case they diverge. Since scour always accompanies acceleration, and deceleration causes the sediment carried by the water to be deposited, it follows that convergent flow must occur in deep spots, and divergent velocities must be correlated with shoals. In fact this was proved by Mr. de Leliavsky's observations and might therefore be taken as the basic empirical law of the natural channel formation in general.

An analogy exists between this reasoning and that which explains the correlation between perpendicular velocity fluctuations in the Prandtl turbulent shear mechanism.[15]

Applying the results of his observations to river-training design, Mr. de Leliavsky found that the groin alternative, wherever applicable, yielded the more satisfactory solution. After his death in 1905 Mr. de Leliavsky's work was continued for some time by I. A. Rosoff, C. A. Akouloff, and N. V. Terpougoff, but results of their work have not been made generally available (1953).

A few words must be added about the constructional merits of the articulated reeinforced concrete bank protection described in the paper, and its substitute, the willow fascine mattress with broken stone as ballast, which was popular in pre-revolutionary Russia.

The latter had a willow skeleton, resembling in shape the inverted ribbed slab of a reinforced concrete floor. This mattress had the advantage of being capable of erection on the surface of the ice in winter, in the immediate neighborhood of the spot where it was to be finally placed. The ice was then removed from this spot, and the completely erected willow structure of the mattress was dragged into the hole thus created and was sunk by loading it with the broken-stone ballast. Surprisingly vast areas could be thus covered in a single operation. Success depended on completion of the sinking within a few hours, thereby preventing water from eroding a hole beneath the partly sunk mattress.

The willow fascine mattress does not belong to either the permeable class or the impermeable class described by the authors, but it is an intermediate, semi-permeable subclass. Its limited capacity to resist percolation of water can be estimated from the fact that 7 in. of head were allowed as a maximum for dams built in this manner.

The great advantage of the willow mattress was that above water level it could rest on almost vertical slopes because the willows soon began to grow and their roots contributed materially to the strengthening of the earthern bank beneath it.

Recent research conducted by the Grenoble (France) laboratory appears to explain from an hydraulic standpoint the structural advantages of such a continuous semi-permeable revetment, as compared with the articulated type that consists of solid elements separated by open articulation joints.

The research at Grenoble was conducted by means of motion-picture studies of reduced scale models of various types of revetments used in North

[15] "The Mechanics of Turbulent Flow," by Boris A. Bakhmeteff, Princeton Univ. Press, Princeton, N. J., 1936, Fig. 30.

African ports. In Fig. 11, let MNP and mnp represent, respectively, the curves of the free wave, and of the forced wave in the granular soil beneath the revetment. The less the permeability of the protection, the greater the difference between the curves.

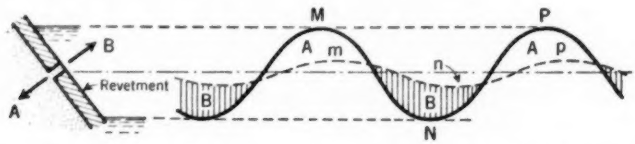



FIG. 11.—FREE WAVES AND FORCED WAVES ACTING ON THE SOIL BENEATH A REVERTMENT

From the results of such tests one may possibly conclude that whenever curve MNP lies above curve mnp, clear water is forced into the joint, as shown by the arrow, A. Under reversed conditions (arrow, B) soil-laden water is pumped out of the joint. Thus, the balance of the soil movements in the vicinity of such a joint is always negative, and eventually this may cause trouble.

The important factor in the design of groins is whether they are built to the high water level, or only to the normal low water level.

## DISCUSSION OF THE DESIGN OF FLEXIBLE BULKHEADS PROCEEDINGS-SEPARATE NO. 166

W. F. Heavey,[3] M. ASCE.—There is no question that there is still a wide range of error in the assessment of the pressures of different soil masses behind bulkheads, despite many theoretical and approximate methods of computing these pressures. The paper by Messrs. Ayers and Stokes does much to illustrate this fact.

[3] Formerly Director, Port of Houston, Houston, Tex.; Project Engr., Frederic R. Harris, Inc., New York, N. Y.

In this connection it is of interest to review the foundation problems encountered in the design and construction of Wharf 16 on the Houston Ship Channel (1951) at Houston, Tex. Borings disclosed that under a layer of very soft organic muck there were several layers of very fine silty sand before silty clay was encountered from El. −40 to El. −60. Below El. −63 a stiff to hard sandy clay-marly with shale was found. Two wharves have slipped out into the channel in the years prior to the construction of Wharf 16 because of these foundation difficulties.

A straight sheet-steel piling bulkhead with two levels of tie-rod supports attached to a sheet-pile anchor wall appeared to be the solution to the problem. The rear sheet-steel piling anchor wall was to extend from El. −26 to El. +9. Construction of this type of anchor wall would not have been so simple as the design indicated because of the inherent difficulty with the two levels of the tie rods and the anchor wall. The problems of bracing and of dewatering the particular site indicated that the cost of this solution would be high. Further study indicated that to assure the effectiveness of the two levels of tie rods, the site would have to be dewatered so that the important lower level of the tie rods could be placed in the dry.

Recourse was taken to changing the design to a series of joined circular sheet-steel piling cells. With this type of piling it was necessary only to remove the layer of organic muck prior to driving the interlocking sheet-steel piling. The 56-ft-wide cells were filled to the top with densified and compacted sands, as was the area between the rear of the cells and the bank to El. +5. Above that elevation the earth was rolled. Care was taken to fill the cells progressively so that the fill inside each cell was always higher than the fill outside it and never more than 5 ft higher than the fill in the adjacent cells.

The sheet piling was driven to El. −53. Each cell acted as a stable unit in resisting both the lateral and vertical loads. As designed, the factor of safety against overturning was 4.0. The factor of safety against sliding along the base of the cell was 5.0, caused by the cohesion of the soil on the shear plane plus the passive resistance of the soil at the toe of the cells on the channel side.

To the valuable axioms listed under the heading, "Axioms for Bulkhead Construction," might be added—where foundation conditions make it economical, the use of circular sheet-steel piling cells, filled with well-consolidated materials, may prove advantageous.

MORGAN S. CAMPBELL,[4] M. ASCE.—The manner in which the authors

[4] Chf. of Design, Corps of Engrs., U. S. Dept. of the Army, Galveston, Tex.

have presented the design problem is to be commended. By carefully avoiding the involved theories of soil pressures acting on flexible bulkheads, they have been able to present an uncomplicated picture of the general problem.

The section entitled "Methods of Reducing Pressures" is interesting and informative, since it describes methods applicable to bulkheads having greater heights and more severe loading conditions than those usually encountered. The "Axioms for Bulkhead Construction" give valuable criteria to be followed in design and construction. To axiom (*e*) might be added an item concerning the possible removal of material from the toe of a bulkhead by scouring action from flowing streams and by propeller wash in boat anchorages and turning basins.

Both concrete and steel sheet-piling bulkheads have been designed and built by the Galveston (Tex.) District of the Corps of Engineers, United States Department of the Army. These walls have ranged in height from 10 ft to 35 ft, and a majority of them have been constructed on navigable waterways. Experience in the design and construction of these walls confirms the design considerations covered by Messrs. Ayers and Stokes.

In the construction of the higher bulkheads in the Galveston District, dikes of oyster shell, similar to the dike shown in Fig. 6(*b*), have been used as the fill behind the bulkhead to obtain reduced lateral pressures. Various types of anchorages, all generally similar to those shown in Fig. 9, have been used with success by the Galveston District. The selection of the type of anchorage has been decided on the bases of economy, availability of materials, and site conditions.

The behavior of bulkheads of steel-sheet piling, when subjected to unusual conditions, and the repair of damage to such structures may be important considerations for some installations. Several of the walls constructed in the Galveston District are located on the Gulf-Intracoastal Waterway, and have been struck at various times by barge traffic. The walls have shown unexpected strength under such collisions. The sheet piling has been bent and flattened, but not badly torn or ruptured at the interlocks, and there has been very little loss of fill material. The part of the piling usually damaged has been between the top of the wall and a point a few feet below the water surface. Repair procedure has consisted of removing the fill immediately behind the damaged section, cutting out and removing the damaged piling, installing new sections of sheet piling above the undamaged portions of the old piling, and splicing the new sections to the old ones. In some instances the old piling has been pulled up several feet to raise the joint above the water before making a welded splice. After the splice has been made, the piling is redriven to grade, wale and tie-rod connections are made, and the backfill is replaced. This method of repair has been found to be economical and it does not cause undue obstruction of the waterway.

WALTER C. BOYER,[5] A. M. ASCE.—The authors are to be commended for

[5] Asst. Prof., The Johns Hopkins Univ., Baltimore, Md.

presenting a valuable addition to the literature concerning flexible-bulkhead design. The determination of the pressure distribution under various conditions of backfill is admittedly complex; hence the classical theories provide a rational solution. The applications of these theories, however, are subject to many variations among designers. Consequently, the statement of the practice of the Bureau of Yards and Docks, which is based on extensive research and experience, will serve as a valuable guide.

It would be helpful if the authors would show the manner in which cohesive resistance and allowance for pore pressure in unconsolidated materials are accounted for in their design procedure. In examples of bulkhead design generally found in the literature, this has been neglected. The work of G. P. Tschebotarioff,[6] M. ASCE, gives promise of yielding important concepts in

[6] "Final Report—Large Scale Earth Pressure Tests with Model Flexible Bulkheads," by Gregory P. Tschebotarioff, Princeton Univ., Princeton, N. J., January 31, 1949.

this matter. There are two methods of accounting for cohesive resistance often employed. One method utilizes a rational weighted value for the angle of friction which is depicted as $\phi_w$ in Fig. 11. The determination of the maximum normal stresses that are developed in the backfill is essential to this method. It is noted that the normal stresses increase with the height of the wall, with a consequent reduction in the value of $\phi_w$. This is, in effect, a rational refinement of the method of selecting a $\phi$-value from a table of soil types—a method found in some specifications. Another method involves the development of pressure diagrams using the value of $\phi$ in the Rankine formula, and subsequently, reducing this pressure by the cohesive resistance $P_c$ defined by the relationship:

$$P_c = \frac{2\,c}{\sqrt{K_p}} \quad \text{.........................} \quad (1)$$

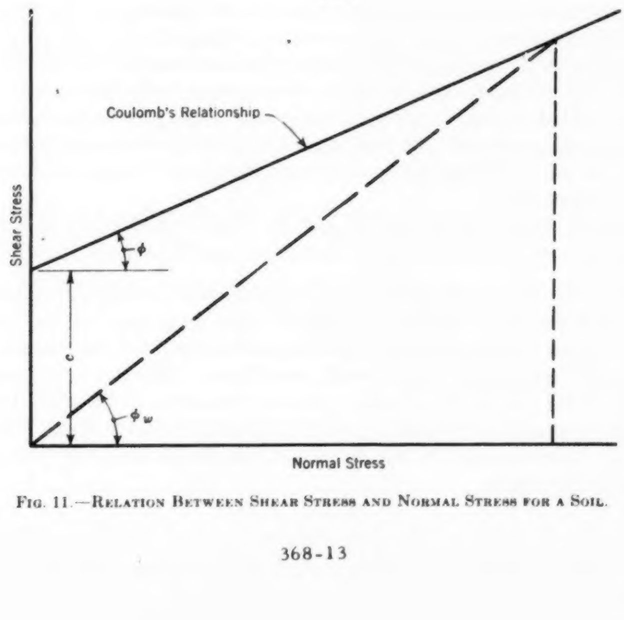



FIG. 11.—RELATION BETWEEN SHEAR STRESS AND NORMAL STRESS FOR A SOIL.

in which $c$ is the cohesion and $K_p$ denotes Rankine's passive coefficient. This may be recognized as the normal modification of the Rankine formula that in-includes cohesive resistance and is utilized occasionally in the design of retaining walls. The pore pressure presumalby can be accounted for by adjusting the conditions of the shear test on which the data for pressure coefficients are based.

The designs of the Bureau of Yards and Docks are "* * * predicated on providing an actual safety factor of from 1.5 to 2.0 for passive resistance." Does this constitute an actual reduction in passive pressure in the computations? When driving in sand, is the Rankine passive coefficient increased by a factor of 2.0, as recommended by H. Blum based on the tests[7] of O. Franzius?

[7] "Versuche mit passiven Druck," by O. Franzius, *Der Bauingenieur*, 1924.

Do the authors consider that the Rankine coefficient, with its neglect of friction between the fill and the wall, actually constitutes a factor of safety?

The methods described for reducing pressures are well conceived and present practical solutions to a number of intricate situations. Although these cover the majority of cases encountered, another situation is worthy of emphasis. The problem of extending harbor facilities by cutting out soil existing at or above water level is occasionally encountered. If the area is excavated, followed by the placing of the wall and the subsequent backfilling, it will fall into the class described by the authors. Another construction procedure for this situation may involve driving the wall and then excavating in front of it. This method is advantageous since it causes little disturbance to the natural soil. If the in-place soil is of a cohesive nature, an economic advantage may be derived by such a construction sequence. In this respect, however, the advice of the authors regarding the development of balanced pressures must be strictly adhered to in the dredging operation.

A bulkhead and relieving platform with the sheet pile located inboard of the platform is shown in Fig. 8. Such a system can cause difficulties if the relieving platform is not surcharged prior to the placing of the hydraulic fill behind the sheet pile. The placing of a surcharge on the relieving platform initially permits interaction of the batter piles and the plumb piles in carrying both the vertical loads and the transverse thrusts. Failure to follow such a sequence of loading may cause poor alinement of the pier face or actual failure of the relieving platform. Examination of Fig. 8 illustrates that the sequence described is the procedure followed by the Bureau of Yards and Docks and is worthy of emphasis.

C. Martin Duke,[8] A.M. ASCE, has described a field study of a sheet-pile

[8] "Field Study of A Sheet-Pile Bulkhead," by C. Martin Duke, *Transactions*, ASCE, Vol. 118, 1953, p. 1131.

bulkhead. An important result of this study is the considerable redistribution of both wall pressure and tie-rod tension when settlement of the backfill occurred. It can be concluded that the support system for the tie rods assumes considerable importance under such conditions. Measures for reducing or eliminating the resulting increases in tie-rod pressures (by placing the rods in conduits) represent additional expense in sheet-pile construction. A description of the measures used by the Bureau of Yards and Docks for this condition would be informative.

The writer is particularly interested in the section entitled "Materials of Construction." Concrete jackets should be used to resist corrosion; however, this is not economically feasible on many installations, with the result that the less expensive bituminous coatings are specified. Experience seems to indicate, however, that even with reasonable care on the part of the contractor, such coatings are far from successful. One of the most pressing needs in flexible-bulkhead construction is the development of an inexpensive coating for sheet piles—a coating which can resist the normal handling necessary on the project and the abrasion to which the wall is subjected during its life.

P. W. ROWE.[9]—A useful record of the practical considerations required for sheet-pile wall construction has been presented by the authors. However, there have been further developments in the knowledge of fundamental factors influencing the stability in cohesionless soil of this type of wall.

[9] Lecturer in Civ. Eng., Univ. of Manchester, Manchester, England.

During the period from 1947 to 1951 a large number of tests[10] were made in Scotland on model sheet-pile walls of varying heights, stiffnesses, and materials. Other variables investigated were the effects of surcharge, dredge level, tie-rod locations, and sand conditions. The results of these tests, which agreed with all other published records of experiments concerned with sheet piling, proved that the bending moment and the tie-rod loads of a sheet-pile wall depend principally on the flexibility of the wall and the density of the subsoil.

[10] "Anchored Sheet-Pile Walls," by P. W. Rowe, *Proceedings*, Inst. of C. E., London, England, Pt. I, Vol. 1, No. 1, January, 1952, p. 27.

Briefly, a stiff wall will be subject to free-earth-support conditions of pressure. If the wall is made more flexible, the pressure distribution will approach that assumed by the fixed-earth-support pressure diagram, and at very high flexibilities the pressure distribution will approach that corresponding to complete fixity. The change is a continuous process, independent of all variables in the design other than the relative density of the subsoil. The results of the investigations conducted in Scotland are shown in the two empirical moment-reduction curves. In Fig. 12 $H$ is the total height of the sheet piling in feet; $E$ denotes the modulus of elasticity of the sheet-pile material in pounds per square inch; $I$ is the moment of inertia of the sheet-pile cross section in inches$^4$ per foot of sheet-pile wall; $D_r$ represents the relative density of the subsoil; $\alpha$ is

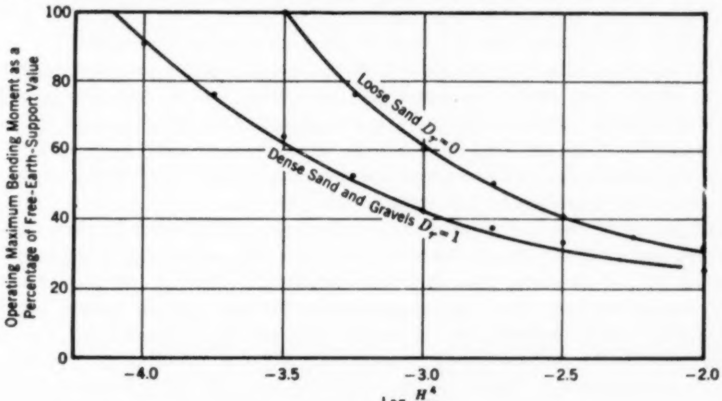



FIG. 12.—REDUCTION OF MOMENT WITH CHANGE IN FLEXIBILITY NUMBER

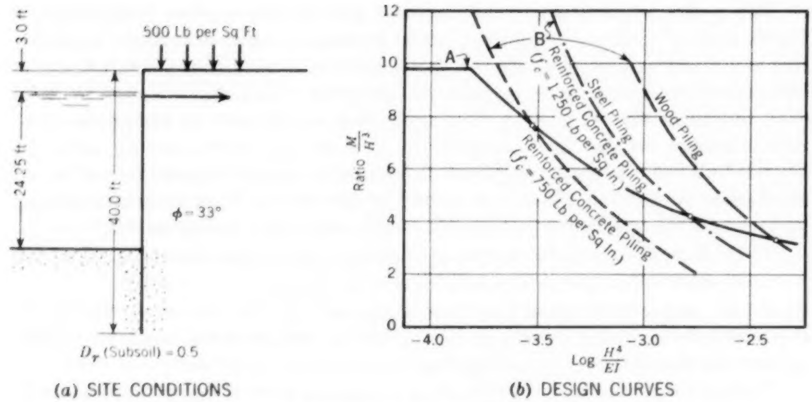



FIG. 13.—THE EFFECT OF MATERIAL OF CONSTRUCTION ON THE DESIGN OF A WALL

the ratio of the free height of the piling to the total length of the piling; and $\frac{H^4}{E\,I}$ signifies the flexibility number of the sheet piling. The curves of Fig 12 are valid for all values of $\alpha$ between 0.6 and 0.8. Theoretical investigations have proven these curves to be correct.

The results of the Scotland investigations signify that the flexibility, and therefore the nature of the material of the piling, must be considered in the design. Specific moment-flexibility curves for different materials of wall are shown in Fig. 13(*b*) which allow the choice of the most economical type of wall and size of section. In Fig. 13(*b*) $M$ is the maximum bending moment on the sheet-pile wall in pound inches per foot of wall length. Curve A was plotted from the maximum free-earth support moment and by interpolating values from Fig. 12 for $D_r = 0.5$. Curves B show the approximate relationship between the unit moment which the particular wall can withstand, and the wall's flexibility number. In the computations for Curves B, the allowable wood fiber stress was 2,000 lb per sq in.; the allowable steel stress was 18,000 lb per sq in.; and the reinforced concrete allowable stresses $f_c$ were 750 lb per sq in. and 1,250 lb per sq. in. In this example the correct design sections are obtained from the intersection of Curve A with appropriate Curve B.

Having determined the required size of the section, the designer generally has to choose the nearest commercially available size. Normally a stiffer section would be chosen. However, it can be seen that the choice of the next lighter section does not lead to the large stress increase as is supposed, since the increase in flexibility of the sheeting causes a further reduction in moment from that corresponding to the design flexibility.

It has been generally assumed (an assumption also endorsed by the authors) that the deeper the piling is driven, the greater the reduction of the bending moment on the sheeting as a result of increased fixity. A slight increase in fixity does occur, but it is a very expensive way of reducing the moment on the sheeting by a few percent. If the wall is stiff and the subsoil loose, the wall would have to be embedded by a length at least as great as its free height before any appreciable fixity commenced. If the wall is flexible and the soil dense, full fixity remains until sudden failure occurs at a penetration of approximately one tenth of the free height. In either case, the earth-pressure

distribution is the free-earth-support diagram at failure. For these reasons the free-earth-support method should be used for the computation of the depth of embedment and the maximum moment acting on a stiff wall. The curves of moment reduction with flexibility for the particular density of subsoil can then be applied to this value.

Under the heading, "Conclusion," the authors state that

> "The most promising avenue to further progress seems to be in the field of prototype measurements for testing the validity of existing design methods."

This suggests that if a wall is designed by a certain method, built, and tested, and it is found to have approximately the design moment, it follows that the design was correct and economical. In fact, if the free-earth-support theory is assumed, the design leads to a stiff section which would cause the prototype to behave in accordance with the free-earth-support computation, and thus result in the computed moment. If the fixed-earth-support theory were assumed, the moment would be less, the section more flexible, and some fixity would result. Probably this theory would also be shown to be valid on the prototype. Thus, no prototype or field measurements on pile walls are likely to be useful until the influence of soil density and pile flexibility is thoroughly understood.

Gregory P. Tschebotarioff,[11] M. ASCE.—A valuable survey of the

[11] Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

various practical considerations which must be evaluated by the designer of flexible anchored bulkheads has been presented by the authors. They are to be complimented for summarizing, in so comprehensive a manner, their extensive experience in this field.

The writer would like to comment on the remarks made by Messrs. Ayers and Stokes concerning the need for preventing high tensile stresses in the tie rods as a result of the consolidation of the backfill. The provision of "adequate vertical supports for the tie rods" (under the heading," Types of Anchorage"), presumably of the type shown in Fig. 9(*a*), may not be sufficient in some cases. Negative friction, caused by the consolidation of the backfill along the skin of such pile supports, can force the piles into the ground and thus overload the anchors which the piles are supposed to support. For this reason, it has been found necessary, in some localities, to encase the tie rods in a hollow box or in a cylindrical conduit so that the fill can settle without loading the tie rods.[12]

[12] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, pp. 511-516.

The authors' recognition of the contributions made by soils engineers to the knowledge of bulkhead design is gratifying. On the basis of the theory of soil mechanics, however, there are several factors that require clarification. For example, no explanation is given as to why the authors state (under the heading, "Soil Pressure Theories") that the "***minimum depth of penetration required for stability of the sheet piles***"—if computed in the manner described—is considered sufficient to give a factor of safety of from 1.5 to 2.0. This condition is apparently attributed to some unspecified properties of "natural undisturbed soils." Other investigators have also used this method of approach—which can lead to overdesign in some cases and underdesign in others.[13] The reasons

[13] "Design Curves for Anchored Steel Sheet Piling," by Walter C. Boyer and Henry M. Lummis, *Proceedings-Separate No. 165*, ASCE, January, 1953.

for this variation are given subsequently.

Model tests have clearly indicated that in the case of sand soils wall friction is fully mobilized below the dredge line in front of the bulkhead. It is this factor which accounts for the very high measured passive resistances.[14] These

[14] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, p. 300.

resistances were, in some cases, three to four times greater than the maximum values theoretically possible if the effect of wall friction is neglected. By thus neglecting the effect of wall friction in computing the passive pressures for granular material, the authors can obtain the desired factor of safety—or even a higher one. For cohesive soils, however, the effect of shearing stresses along the wall on the increase of passive resistance will be much smaller than in the case of granular materials. Therefore, by neglecting the effect of wall friction (or adhesion), and by using the minimum depth of penetration required for stability of the sheet piles, a factor of safety may be obtained for clay soils that is less than the minimum value of 1.5.

Another matter of both theoretical and practical importance which was not entirely clarified concerns the criteria that should be used to decide whether and when the depth of penetration should be increased beyond the depth necessary for stability, so that the pile point can be considered to be restrained and the fixed earth support method of design can be used instead of the free earth support method.

Both of these factors—questioned because they lead to ultra-conservative designs in some cases of granular soils—have been discussed by the writer.[15]

[15] Discussion by Gregory P. Tschebotarioff of "Design Curves for Anchored Steel Sheet Piling," by Walter C. Boyer and Henry M. Lummis, *Proceedings-Separate No. 368*, ASCE, December, 1953.

The writer agrees with Messrs. Ayers and Stokes that much information may be gained from prototype measurements, testing the validity of design methods. The importance of such measurements has long been recognized; however, at this time (1953) the results of only one such complete set of measurements on anchored bulkheads have been published.[8] Opportunities for field observations have been by-passed because of (1) a deficiency of funds for such observations, (2) the lack of specially trained personnel to conduct such observations and to be responsible for them, and (3) the wide-spread feeling among engineers responsible for waterfront construction that allocation of funds from a specific project is not justified unless the observations can be relied on to produce results of value to that particular project.

This latter feeling is responsible for the fact that years go by with nothing being done to check the unsatisfactory methods of design in use. It is therefore essential that funds be appropriated for such observations and that these observations be placed in the hands of specialized research personnel.

It should be remembered that no matter how well full-scale field observations are organized, such observations alone cannot form the basis for a rapid advance of knowledge of the actual performance of earth-retaining structures. The number of conditions affecting the performance of such structures is so great that (in accordance with the theory of probability) there is little chance that these factors would vary from one observed structure to another in a manner that would permit the numerical evaluation of the separate influence of each factor. In order to accomplish this evaluation under controlled conditions, model testing is essential.

The authors present an example of the important results achieved by model testing. The laboratory experiments that they state as having justified the use of sand dikes to reduce the pressures of fluid hydraulic fill were model tests performed under the sponsorship of the Bureau of Yards and Docks.[16] The

[16] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, pp. 293-297.

writer, in this discussion, presents another example of the importance of model test results in their effect on the estimation of the factors of safety for a known depth of sheet-pile penetration.[14, 16]

Further progress can therefore be best achieved by a continuous effort to correlate the results of full-scale observations, model tests, and theoretical analyses.[17]

[17] "Some Unsolved Problems of Importance for the Design of Earth Retaining Structures," by Gregory P. Tschebotarioff, *Bulletin No. 33*, Permanent International Assn. of Nav. Congs., 1950.

J. Owen Lake,[18] A.M. ASCE.—A useful summary of the various factors

[18] Cons. Engr., Toronto, Ont., Canada.

to be considered in the design of bulkheads has been presented by the authors. Unfortunately, however, the description of earth-pressure computation (under the heading, "Soil Pressure Theories") is not sufficiently explicit to allow detailed discussion. It would be enlightening if the authors' method of determining the active and passive earth pressures for silts and clays could be explained.

Messrs. Ayers and Stokes state that earth pressures for fine-grained and cohesive materials are based on both frictional and cohesive properties. However, soils are shown in Figs. 3, 5, and 10 with $\phi$-values of 14°, 18°, and 26° with no mention of cohesion and no indication on the loading diagrams to show that cohesion is allowed for. Because soils having values of $\phi$ that are less than approximately 30° inevitably possess cohesion, neglect of this property will nullify the validity of any pressure computation. Also, an analysis of the initial stability of bulkheads sustaining pressure caused by fully saturated clays should be based on the clay possessing a $\phi$-value equal to zero, as this is the result obtained in undrained triaxial tests, which generally represent immediate post-construction conditions before any water content change has occurred.

The authors compute the active and passive pressures of granular materials acting on bulkheads by the Rankine-Coulomb formulas, neglecting the effect of wall friction, and then apparently design the bulkhead for simple support. Such a procedure will generally overestimate the required strength of the sheet piling by approximately 25% to 50%, in comparison to a design that uses the fixed earth support method[19] based on angles of wall friction equal to zero for

[19] "The Application of Steel Sheet Piling to Engineering Construction," by S. Packshaw, *Civil Engineering*, London, England, December, 1932—January, 1933.

active pressure, and equal to $\frac{2}{3}\,\phi$ for passive pressure. This latter procedure has been the basis of design adopted by the writer for considerable bulkhead construction in the United Kingdom and in many other parts of the world, and at least some confirmation of the general validity of this design procedure lies in the fact that there has been no indication that stress in these bulkheads has exceeded the anticipated design value of $\frac{1}{2}$ the yield stress.

The generally satisfactory results obtained from the fixed earth support method when applied to bulkheads founded in cohesionless soils are made apparent in the results of model tests described by Mr. Rowe,[20] who has

[20] "Anchored Sheet-Pile Walls," by Peter W. Rowe, *Proceedings*, Inst. of C. E., London, England, January, 1952.

presented, for the first time, quantitative information concerning the influence of bulkhead flexibility on the maximum bending moment which bulkheads sustain. In a discussion of Mr. Rowe's tests, S. Packshaw and the writer have shown comparisons between typical designs based on the conventional fixed earth support method with no allowance for flexibility, and designs based on the results of Mr. Rowe's research.[21] It was demonstrated that generally

[21] Discussion by S. Packshaw and J. Owen Lake of "Anchored Sheet-Pile Walls," by Peter W. Rowe, *Proceedings*, Inst. of C. E., London, England, September, 1952.

even the fixed earth support method considerably overestimates the bending moment. The comparisons did disclose that sheet-pile bulkheads of normal flexibility having a yield stress of about 36,000 lb per sq in., penetrating into loose cohesionless soil and exceeding approximately 40 ft in height, did not achieve the degree of fixity assumed in the fixed earth support method. However, the existence of uniform deposits of entirely cohesionless sand in a minimum state of compaction, although not unknown, is not often encountered in bulkhead construction. Furthermore, the use of high-tensile steel sheet piling would generally develop the necessary flexibility at ½ the yield stress to achieve fixity even in loose sand.

Thus, the use of high-tensile steel having a yield stress of from 50,000 lb per sq in. to 55,000 lb per sq in. can, under appropriate conditions, result in economy amounting to about 35% saving in weight and 25% saving in cost when compared with the use of steel having a yield stress of approximately 36,000 lb per sq in.

Although the authors consider that the greatest water depth for conventional anchored bulkheads set in granular material is about 30 ft, it is of interest to note that the writer has been concerned with a design involving 42 ft of water, and a conventional bulkhead with a single anchorage system located at water level was adopted with success. In this design the steel sheet piling was a standard section having a section modulus of 55.1 cu in. per ft of wall, but strengthened in the region of maximum bending moment to result in a section modulus of 82.2 cu in. per ft of wall. Similar designs have been proposed and adopted in Canada, with resulting economy.

Nai C. Yang,[22] A. M. ASCE.—The writer takes issue with the authors'

[22] Foundation Engr., Madigan-Hyland, Cons. Engr., Long Island City, New York, N. Y.

statement (under the heading, "Soil Pressure Theories") that

> "By use of the pressures computed as outlined above [Rankine-Coulomb formulas], the minimum depth of penetration required for stability of the sheet pile is determined. This minimum depth is considered sufficient to give the required factor of safety for sheet piling driven into natural undisturbed soils."

It is known that the magnitude of earth pressure is related to the amount of wall movement. This movement indicates the mobilization of the shearing strength in the soil. The conventional earth pressure theory assumes that the shearing strength of the soil is fully mobilized to create the active or passive stress. In so far as the state of stress is concerned, the active pressure computed by the Rankine-Coulomb formulas is the minimum thrust on the wall, and the passive pressure is the maximum resistance that the soil can afford.

In known cases of bulkhead failure, the wall moves outward near the dredge line a distance more than the movement at the anchorage. This type of wall movement prevents the soil from fulfilling the basic assumption of the Rankine-Coulomb theory. For active earth pressure, the pressure shifts toward the anchor point at which the anchorage confines the wall movement. According to reported field measurements,[23] the observed active pressure does not differ

[23] "Earth Pressure Measurements in Open Cuts," by R. B. Peck, *Transactions*, ASCE, Vol. 108, 1943, pp. 1008-1036.

greatly in the total thrust from the pressure as computed by the Coulomb formula. It can be assumed that the ordinary wall movement might create an active pressure state in the backfill. Coulomb's formula should be used in designing the bulkhead only to estimate the magnitude of the total active thrust, not its distribution on the wall.

To create the maximum passive resistance, the wall must move more than the distance that is required to develop the active thrust. Although the backfill reaches a state of active pressure, the same wall movement is not sufficient to mobilize fully the passive resistance of the front soil. The actual passive resistance is thus appreciably smaller than the theoretical resistance. The dredge line is sometimes assumed to extend horizontally from the bulkhead. If this were the case, there would be an unrealistic increase in computing the passive resistance. With the dredge line set at a slope equal to the angle of repose of the soil, there will theoretically be no passive resistance at all.

If the limitations of earth pressure theory are ignored, the bulkhead design by the Rankine-Coulomb formulas eventually will be found to be unstable. In most cases, bulkhead failures have resulted when the bulkhead collapsed massively along a potential sliding surface somewhere under the sheeting. The second major cause of failure was the inadequacy of the anchorage. This inadequacy tended to make the wall tilt. There have been few cases in which the sheet pile broke near the dredge line where, according to the conventional earth pressure theory, the bending moment should be a maximum.

In designing bulkheads, it should be noted that:

1. The stability of the entire bank is the most important aspect to be investigated. The penetration of the sheet pile should meet the requirement for slope stability.

2. A study of the possible movement of the anchorage should be more important than that of its potential resistance.

3. The earth pressure on the sheeting is of relatively minor importance. The bending moment of the sheeting can be estimated, however, by use of the conventional earth pressure theory. Any such computation is of only academic interest unless the primary assumptions of the theory are fulfilled.

*Hydraulic Backfill.*—On the basis of experience the most desirable site for the use of hydraulically-placed backfill is in shallow water. The soil condition of the foreshore usually consists of a fine-grained deposit that has never been subjected to a pressure greater than its prevailing overburden. Such a deposit is usually referred to as being normally loaded, and it is always soft to a considerable depth below the surface. The hydraulic fill generally affords an economical way for the disposal of the dredged materials. It is not uncommon for the hydraulic fill to be silted upon soft virgin ground. For sheet-pile bulkheads with hydraulic backfill, experience indicates that: (1) Failure occurs

more frequently than for bulkheads with dry backfill; (2) sheet piles are usually broken near the anchor point; and (3) many filling enclosures fail to stand at the outboard corners.

Excessive water pressure in the fill is usually thought to be the immediate cause of bulkhead failure. The authors (under the heading, "Methods of Reducing Pressures") have presented their experiences concerning "The effect of a sand dike in reducing the lateral pressure * * *." The application is limited to the condition that "* * * the existing material is assumed to be of good quality at the level of the dredged bottom, overlain by a few feet of somewhat poorer material." However, he realizes that "Frequently, the site for a bulkhead is overlain by a considerable depth of soft material." It is unlikely that the concept of a sand dike can be used extensively in hydraulic-filling operations.

Because of the weight of the fill, the subsoil tends to consolidate. According to the elastic theory of settlement[4], the edges of a large-area filling basin should subside about one half the amount that the fill subsides, and the corners should subside approximately one quarter of the amount. The differential settlement at the edge is about one half the total settlement of the fill, and the differential settlement at the corners is three quarters of the total settlement. It is important that excessive settlement is always anticipated in a normally loaded deposit. In the case of sheet-pile bulkhead with hydraulic backfill, the anchorage should settle more than the sheeting. Because of the differential settlement, the anchorage tends to pull the sheeting against the fill. This type of wall movement contradicts the fundamental assumption (in conventional bulkhead design) that the sheeting tends to pull the anchorage. The backfill consequently by-passes the active state of earth pressure and reaches the passive one. The tie-rod tension and the bending moment in the sheeting increase considerably—as the passive earth pressures do with increasing wall movement. The more differential settlement that occurs, the more possibility there will be for bulkhead failure.

If the fill is assumed to be cohesionless, sandy soil with $\phi = 30°$ and a saturated weight of 115 lb per cu ft, according to the Coulomb formulas the active pressure for dry fill is 38 lb per sq ft, the active pressure for hydraulic fill equals 80 lb per sq ft, and the passive pressure for hydraulic fill is 220 lb per sq ft.

The active pressure in the hydraulic fill is approximately twice that in a dry fill. By changing the state of stress, however, the earth pressure may increase as much as six times. In usual design practice, the working stress of a structural material is taken to be one third of its stress at failure; it is therefore unlikely that a stress that is twice the working stress will cause an immediate failure. These numerical quantities are purely illustrative. They show that, unless there is a change of state of stress (from active to passive), the excessive water pressure is not principally responsible for the failure of a bulkhead with hydraulic backfill.

The sand dike does reduce the excessive water pressure on the sheeting. Nevertheless, a preconsolidation of the area along the fill edges will result from the placement of the sand dike. As the hydraulic fill is placed, the differential settlement between the sheeting and the anchorage will increase. Thus, the advantage of using a sand dike is not offset by its potential harm.

DAVID HOPKINS,[24] A.M. ASCE.—A clear and concise summary of basic

[24] Civ. Engr., Parsons, Brinckerhoff, Hall & Macdonald, New York, N. Y.

principles has been presented by the authors. The paper will undoubtedly be of assistance to engineers responsible for bulkhead design. It is regrettable that references to earth-pressure analyses and research performed by Mr. Terzaghi and several others[10,25,26,27,28,29,30] have not been included.

[25] Correspondence by G. P. Tschebotarioff on "Anchored Sheet-Pile Walls," by Peter W. Rowe, *Proceedings*, Inst. of C. E., Pt. 1, Vol. 1, 1952, p. 616.
[26] Correspondence by Karl Terzaghi on "Anchored Sheet-Pile Walls," by Peter W. Rowe, *ibid.*, p. 619.
[27] Correspondence by Savile Packshaw and J. Owen Lake on "Anchored Sheet-Pile Walls," by Peter W. Rowe, *ibid.*, p. 621.
[28] "Large-Scale Model Earth Pressure Tests on Flexible Bulkheads," by G. P. Tschebotarioff, *Transactions*, ASCE, Vol. 114, 1949, p. 415.
[29] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 216.
[30] "Earth-Pressures on Flexible Walls," by J. P. R. N. Stroyer, *Journal*, Inst. of C. E., Vol. 1, 1935–1936, pp. 94, 550.

The conclusions reached by model investigators and research workers should lead to considerable modification of the design criteria used by the Bureau of Yards and Docks. Present knowledge (1953) indicates that certain design criteria in use by the Navy are ultra-conservative and wasteful.

*Design Principles.*—Publications describing model tests have been confusing because their writers have advocated conflicting design methods for solving bulkhead design problems. J. P. R. N. Stroyer, for example, attributed an observed reduction in bending moment to soil arching, and he derived an empirical formula to evaluate this reduction.[29] Research by Mr. Rowe illustrated the fact that the bending moment reduction is only partly a result of soil arching; the major causes are flexure of the piling and fixity of the toe.[10]

Despite such disagreement as to the best method of bulkhead design, the results obtained from the various model tests are substantially in agreement. A number of basic principles are well established, which can be briefly summarized as follows:

1. Active and passive pressure. The active and passive pressures of a granular soil on a flexible bulkhead generally follow the Coulomb theory with full wall friction. The magnitude and distribution of the earth pressures, however, and the bending moment in the sheet piling, will depend on (*a*) the amount of yield of the tie rod under load, (*b*) the flexure of the sheet piling, and (*c*) the density of the soil which provides the toe embedment for the sheet piling.

2. Tie-rod yield. The amount of tie-rod yield and anchorage yield under load has considerable influence on the magnitude and distribution of active pressure. In the hypothetical case of a rigidly-supported rigid bulkhead, the active earth pressure would be equal to or greater than the earth pressure at rest. Since cases of zero yield cannot occur in practice, however, earth pressure at rest need not be considered.

When the tie rod yields, the active earth pressure decreases until it becomes approximately equal to the Coulomb value—but with a soil-arching effect which results in a pressure redistribution, as illustrated in Fig. 2(*a*). If the tie-rod yield increases, the soil-arching effect disappears, and the active pressure follows a straight line distribution equal to, or slightly less than, the Coulomb value with full wall friction.

Thus, for the range of yield normally encountered in tie rods and anchorages, the active pressure—may be assumed—without serious error—to be equal to the Coulomb value with full wall friction. However, if the tie-rod yield is restricted, as in the case of a stiff batter pile anchorage with a very short tie rod, there will be a concentration of pressure at the tie-rod level, and a resulting increase in tie-rod pull. As a general rule, however, yield of the tie rod has the effect of slightly relieving the active pressure on the bulkhead.

3. Flexure of the piling. Flexibility of the sheet piling is the most important factor contributing to the active-pressure reductions and bending-moment reductions observed by investigators. The causes of the reduction are complex, but they are generally related (*a*) to the amount of end fixity provided by the embedment of the toe of the pile, and (*b*) to the reduction in effective span which occurs if the pile is flexible. In the case of a sheet pile of average stiffness, where the toe has a good penetration in a dense soil, the negative bending moment at the toe will approximately agree with classical fixed-earth-support computations, and there will be a corresponding reduction in the positive moment at midspan.

If the sheet-pile section is very stiff, however, and the toe is embedded in a loose soil, the toe of the pile may not be fixed, even though the pile has been driven to a penetration that should theoretically provide end fixity. In this case, the positive bending moment may be equal to the moment in a simply-supported beam spanning from tie-rod level to the centroid of the passive pressure. The bending moment in this extreme case may be double or more than double the bending moment for the preceding case.

4. Soil density and passive-pressure distribution. The effects of pile flexure and the soil density are closely related in the determination of the toe fixity. The distribution of passive pressure is similarly affected by both pile flexure and the soil density. However, the full Coulomb passive pressure is not usually mobilized. Only in the case of a sheet pile driven to a minimum penetration to resist sliding is the magnitude and the distribution of the passive pressure exactly equal to the Coulomb value.

The possible variations in the passive pressure are illustrated in Fig. 14, which shows three typical cases. The active pressure on the sheet piling is assumed to be the same in each case. In Fig. 14 the effective span is denoted by $L_e$.

Fig. 14(*a*) shows a pile driven to a minimum penetration, with the passive pressure $P_1$ equal to the Coulomb passive pressure. The minimum penetration varies from one-fifth to one-third of the pile length in an average bulkhead design, depending on the $\phi$-value and other design factors.

Fig. 14(*b*) shows a very stiff pile driven to a greater penetration in a loose soil, where the full passive pressure is not mobilized. The passive pressure $P_2$ is slightly greater than $P_1$, and the bending moment is larger because of the increase in effective span.

Fig. 14(*c*) shows an extremely flexible pile driven into a dense soil. The full passive pressure is mobilized, but it does not follow a straight line distribution because of the pile contraflexure. In this case, $P_3$ is approximately equal to $P_1$ plus $P_4$. The bending moment in the pile is reduced by the amount of the end-fixing moment.

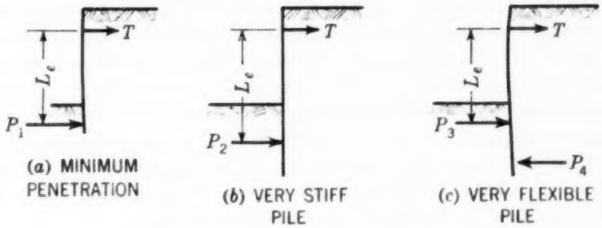



FIG. 14.—PASSIVE-PRESSURE DISTRIBUTION

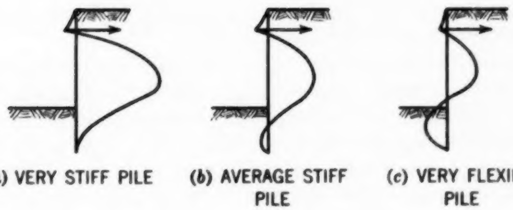



FIG. 15.—BENDING-MOMENT DIAGRAMS FOR BULKHEADS IN VERY LOOSE SOIL


FIG. 16.—BENDING-MOMENT DIAGRAMS FOR BULKHEADS IN DENSE SOIL

*Design Analyses.*—The most comprehensive series of model tests of flexible bulkheads is the investigation undertaken by Mr. Rowe in 1952.[10] The results of his measurements are shown in an abbreviated and simplified form in Figs. 15 and 16, which illustrate the effect of pile flexure and soil density on the bending moment of the pile. It can be seen how critical these factors are, and how wide a range of bending moments can occur under varying conditions.

A survey of Mr. Rowe's results shows that for a pile of average stiffness in a soil of average density (Fig. 15(*b*)) the bending moment is of the same order of magnitude as the bending moment computed by the conventional fixed earth support method. If a very stiff pile has a toe embedment in a loose soil (Fig. 15(*a*)), the bending moment may be considerably larger. For a very flexible pile in a dense soil (Fig. 16(*c*)) the bending moment may be smaller than that computed by the fixed earth support method.

It should be noted that in the following arbitrary pile classification (according to varying degrees of stiffness), it is assumed that (1) piles of average stiffness include steel sheet piles in the lengths normally used for anchored sheet-pile bulkheads; (2) very stiff piles include reinforced concrete sheet piles and short lengths of the heavier sections of steel sheet pile; and (3) very flexible piles include timber sheeting and longer-than-average lengths of steel sheet piles.

As an example of relative stiffness, a pile 50 ft to 55 ft long with an MZ 38 section would be typical of piling having an average stiffness, whereas a similar section 30 ft to 35 ft long would be very stiff.

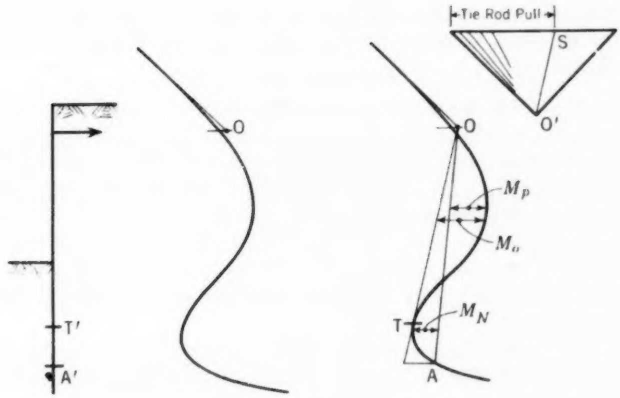
It should also be noted that the higher the working stress is, the greater the pile flexure will be. Hence, overstressing a pile in bending will automatically relieve the pressure on it, and consequently reduce the bending moment.

It is unfortunate that Mr. Rowe's method for design analysis is unwieldy and pre-supposes more exact information about soil properties than is usually available to the designer. It is clear, however, that an allowance for bending moment variations must be included in practical design computations, and the most convenient method of making the necessary adjustments to design moments appears to be an empirical adaptation of the fixed earth support analysis.

The following graphical procedure (which is approximate, but practical) has been found to be in reasonable agreement with more rigorous analytical methods:

(*a*) The active and passive pressures are computed using full wall friction equal to $\frac{2}{3}\,\phi$. To avoid arithmetical work in the computations, the Rankine values of active pressure can be used—neglecting wall friction—and the error will be small. In computing passive pressures, wall friction must be included, and the passive pressure can be twice the Rankine value or more. Values of $K_p$ can be obtained from tables or graphs to include wall friction. It is also important that account be taken of the effects of hydrostatic or pore pressures as a result of tidal fluctuations or ground water on the pressure diagram.

(*b*) The force polygon is drawn from the vector diagram. There is no need to locate the closing line precisely by the elastic line method, however, since its position depends on the degree of fixity of the toe, and is extremely variable.




(a) FORCE POLYGON (b) MOMENT DIAGRAM

Fig. 17.—Graphical Analysis

In Fig. 17(a), the force polygon has been drawn, and the closing line, which will constitute the base line of the bending moment diagram, will pass through point O (point O lies on the line of action of the tie rod). The base line will actually lie somewhere in between line OT (which is tangential to the force polygon), and line OA (which is drawn so that $M_n = M_p$). If there is no fixity at the toe, the maximum will be $M_O$; if the toe is completely fixed, the maximum value will be $M_p$ (which is equal to $M_n$).

(c) The tie-rod pull is obtained from the vector diagram by drawing a vector O'S parallel to line OT, and measuring the resultant force. The value of the tie-rod pull obtained by this procedure (assuming that there is no toe fixity) is a maximum. In most cases where toe fixity occurs, the tie-rod pull is less, but by using the maximum figure an adequate safety factor is automatically introduced into the design. No safety factor is required other than the use of a normal working stress for the tie rod—20,000 lb per sq in. if structural steel is used.

(d) The required penetration of the toe of the pile is determined after the amount of toe fixity has been estimated. The toe penetration depends on (1) the density of the soil and (2) the section modulus of the sheet piling. Sand and gravel can generally be considered to be dense, unless it is from a recently deposited bed or in a loose, cohesionless state.

If it is proposed for economic or other reasons to use a sheet pile with a section modulus adequate for the maximum bending moment, the toe can be cut off at the elevation of the point T'. Alternatively, if the soil is dense enough to develop full toe fixity, a lighter section of sheet pile can be used; the toe penetration is then extended to the elevation of point A. In order to provide a reasonable margin of safety against toe failure, the actual length of pile must be increased so that the penetration is least 20% more than the minimum penetration found by this method. (For an average bulkhead design this requires a pile length approximately 5% greater than the theoretical length.)

(e) For piles of average stiffness, the positive moment $M_p$ can be used to obtain the required section modulus (assuming that $M_n$ equals $M_p$, as shown in Fig. 17(b)).

For very flexible sheet piling, the actual bending moments will be smaller than $M_n$ or $M_p$, but it is not advisable to make any further reduction in the section modulus of the sheet pile without a very thorough investigation of soil properties and a rigorous analysis of earth pressures and moments for the particular pile section that it is proposed to use.

For very stiff piles or for piles embedded in a loose soil, the bending moment can be taken as the full positive value without end fixity ($M_O$ in Fig. 17(b)). In the case in which the density of the soil is doubtful, a value of $M_n$ equal to $\frac{1}{2}$ $M_p$ can be used.

In all cases a working stress of 24,000 lb per sq in. or greater can be used for regular sheet-pile sections, and the factor of safety will be adequate if the soil properties have been reasonably estimated. If severe corrosion is anticipated, it is necessary either to increase the thickness of the steel section or to resort to cathodic protection.

*Conclusions.*—In conclusion, the following suggestions are advanced:

1. Consolidation and correlation of existing knowledge are more important than further model research. In particular, a convenient method for evaluating pile flexibility and soil density and their influence on pile bending moments is urgently required.
2. A full-scale experimental verification of pile flexibility effects is required to support model test data.
3. Free earth support analyses which compensate for toe fixity by including a bending moment reduction factor are liable to be misleading; fixed earth support methods should always be used.
4. Design analyses should be suitable for practical design use. In view of the approximations involved in "idealizing" geologic sections and assessing soil properties, design computations should not depend on arithmetical accuracy to several decimal places.

## PROCEEDINGS-SEPARATES

## VOLUME 79 (1953)

The technical papers published in the current calendar year are presented below. Technical division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering."

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AUGUST: 230(HY), 231(SA), 232(SA), 233(AT), 234(HW), 235(HW), 236(HW), 237(AT), 238(WW), 239(SA), 240(IR), 241(AT), 242(IR), 243(ST), 244(ST), 245(ST), 246(ST), 247(SA), 248(SA), 249(ST), 250(EM),[c] 251(ST), 252(SA), 253(AT), 254(HY), 255(AT), 256(ST), 257(SA), 258(EM), 259(WW).

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OCTOBER:[d] 290(all Divs), 291(ST),[c] 292(EM),[c] 293(ST),[c] 294(PO),[c] 295(HY),[c] 296(EM),[c] 297(HY),[c] 298(ST),[c] 299(EM),[c] 300(EM),[c] 301(SA),[c] 302(SA),[c] 303(SA),[c] 304(CO),[c] 305(SU),[c] 306(ST),[c] 307(SA),[c] 308(PO),[c] 309(SA),[c] 310(SA),[c] 311(SM),[c] 312(SA),[c] 313(ST),[c] 314(SA),[c] 315(SM),[c] 316(AT),[c] 317(AT),[c] 318(WW),[c] 319(IR),[c] 320(HW).[c]

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a. Beginning with "Proceedings-Separate No. 200," published in July, 1953, the papers were printed by the photo-offset method.

b. Presented at the Miami Beach (Fla.) Convention of the Society in June, 1953.

c. Presented at the New York (N.Y.) Convention of the Society in October, 1953.

d. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.

e. Discussions, grouped by divisions.